Preliminary analysis of instrumented Wellington building responses in the July/August 2013 Seddon/Lake Grassmere earthquakes

2014 NZSEE Conference

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ABSTRACT: This paper investigates the dynamic response of five instrumented structures in the greater Wellington region during the July/August 2013 Seddon/Lake Grassmere earthquakes. Six events ranging from M_w 5.6 - 6.6 were considered in the analysis of five different structures: GNS Science at Avalon, BNZ Centre Port, Majestic Centre, Victoria University Te Puni Village Tower, and Wellington Hospital. This preliminary investigation firstly involved determination of the fundamental period of each structure in the principal horizontal directions, and comparison with the simplified method proposed in NZS1170.5:2004. Secondly, the peak floor level acceleration distributions over the height of the structures were analysed and compared with the design provisions in NZS1170.5:2004 and ASCE/SEI7:2010., The results illustrate that the simplified equation for fundamental period in NZS1170.5:2004 often underestimated the fundamental period and could result with unconservative design forces when using forced based design, and that both NZS1170.5:2004 and ASCE/SEI7:2010 provisions for floor acceleration contain significant deficiencies due to the omission of the dependence of peak floor acceleration variation with height on vibration period.

1 INTRODUCTION

In 2007, GNS Science, with funding from the Earthquake Commission, embarked on a programme to instrument various structures throughout New Zealand to capture the seismic response when subjected to a ground motion excitation (Uma, Cousins, & Baguley, 2010a). Such information is fundamental for engineers to assess the validity of the underlying assumptions used in structural design and analysis.

The GeoNet Building Instrumentation Programme selected various structures for instrumentation based off their proximity to a fault, and importance with regard to functionality and economic significance. Additionally, the instrumented structures are intended to be representative of New Zealand's building stock or utilize unique structural systems, such as base isolation or sliding hinge joints (SHJ) (Uma et al., 2010a). Currently there are ten structures with completed monitoring arrays, six of which are in the greater Wellington region, and five of which are the focus of this paper.

The events of the Canterbury earthquake sequence enabled the first detailed analysis of a New Zealand instrumented structure subjected to strong ground motion loading (McHattie, 2013). However, due to the paucity of earthquakes producing strong ground motions in Wellington, a large scale analysis of the dynamic response of multiple New Zealand structures had not been possible, until the July/August 2013 Seddon/Lake Grassmere earthquakes.

The July/August 2013 Lake Grassmere earthquake sequence produced two notable events which resulted in peak ground accelerations (PGAs) in excess of 0.1g on both sides of Cook Strait (GNS Science, 2013). These two events and other aftershocks generated data that enabled a detailed analysis of five buildings in the Wellington Region which are examined in this paper.

2 EARTHQUAKE EVENTS CONSIDERED

The six largest events (in terms of ground shaking in Wellington) from the 2013 Lake Grassmere earthquake sequence were selected for analysis as presented in Table 1. The events range in magnitude from M_w5.6-6.6 and produced PGA values ranging from 0.015-0.11g at the Wellington Te Papa Museum (TEPS). Although all six events were considered, the two largest magnitude events were given the greatest consideration as they induced the largest seismic responses in the instrumented structures and therefore are the most useful for analysis purposes.

Table 1. Summary of earthquake events considered in analysis of instrumented structures. PGA's recorded at Te Papa (TEPS) are indicative of the shaking in the Wellington region

Magnitude (M_w)	Event date and time	PGA at TEPS (g)
5.7	18/07/2013, 2106	0.019
5.8	20/07/2013, 1917	0.026
6.5	21/07/2013, 0509	0.11
6.6	16/08/2013, 0231	0.091
5.6	16/08/2013, 0351	0.015
6.0	16/08/2013, 0531	0.015

3 INSTRUMENTED BUILDINGS EXAMINED

The GeoNet Building Instrumentation Programme has active accelerometer arrays in ten structures located throughout New Zealand. Six of these instrumented structures are located in the Wellington region. Instruments were installed in these structures due to their location in an area of high seismicity. Of these six structures, five were selected for further analysis, the properties of which are summarized in Table 2. The sixth structure, the Thorndon Overpass bridge, was omitted as the dynamic response of buildings is the primary focus of this paper. The instrumentation arrays consists of accelerometers located strategically throughout each building to capture critical seismic response phenomena such as higher mode effects, torsional oscillations and foundation rocking (McHattie, 2013). The instruments record accelerations in three orthogonal directions.

Table 2: Summary of the five structures analysed for multiple earthquake events

Building name	Station code	Height (m)	No. of stories	Construction year
GNS Science Avalon	AVAB	13	4	1973
BNZ Centre Port	CPLB	27	6	2009
Majestic Centre	MJCB	116	29	1991
Victoria University Te Puni Village	VUWB	37	10	2008
Wellington Hospital	WHSB	30	6	2007

3.1 GNS Science, Avalon

The structural layout consists of two rectangular units of approximately equal dimensions running perpendicular to each other separated by seismic gaps. The structure was selected for instrumentation to represent a low-rise reinforced concrete (RC) moment resisting frame (MRF) building constructed prior to 1976 (Uma et al., 2010a). The first unit has only instrumentation on two levels located along a vertical array, while the second unit has it on three levels. Only the results for the second unit are included in this paper.

3.2 BNZ Centre Port

The building is situated on reclaimed land in Wellington Harbour and is comprised of three five storey piers linked together by pedestrian bridges across two atria. The lateral load resisting system is MRFs constructed in RC. A proprietary RC hollow core flooring system gives clear spans of 17m which is substantially larger than most office buildings. The instrumentation array consists of 14 accelerometers primarily located on various levels in the southwest pier of the building. (Uma, Cousins, Young & Baker, 2010d). This building suffered extensive non-structural damage from the 21st of July 2013 earthquake (Dominion Post, 2013).

3.3 Majestic Centre

The Majestic Centre is the tallest office building in Wellington at 116m high. The lateral load resisting system is constructed in RC and consists of a central shear core with perimeter MRFs (Uma, Nayyerloo, Cousins, & Young, 2013). The structure is currently undergoing seismic retrofit with the target of 100% new building standard (NBS) (KIPT, 2013). In total 15 instruments are spread throughout the structure with a series of instruments in vertically aligned with the core of the building on levels 10, 20 and 29. (Uma et al., 2013).

3.4 Victoria University Te Puni Village Tower

This ten storey structure is part of the larger Te Puni Village complex which encompasses three main buildings. The lateral load resisting system comprises of lightweight steel MRFs with SHJs for additional energy dissipation. 12 accelerometers are distributed throughout the structure, eight of which are in vertical alignment. (Uma, Cousins & Young, 2010b).

3.5 Wellington Hospital

Wellington hospital is an importance level 4 (IL4) structure designed to be fully operational immediately preceding a maximum credible earthquake (MCE) event. The structure is base isolated with a combination of lead rubber and slider bearings. The layout of the structure is irregular comprising of multiple wings protruding from a central core. The instrumentation array consists of 16 accelerometers primarily located on perimeter walls with at least one instrument per level of structure. (Uma, Cousins & Young, 2010c).

4 FUNDAMENTAL PERIOD DETERMINATION

A critical structural property used in design and analysis is the fundamental period. Accurate estimation of this period is essential in order to predict the dynamic characteristics of a structure and thus deformations and forces that it will be subjected to. Two methods exist for obtaining the fundamental period based off observed data. These are Fourier and Response spectral ratios (McHattie, 2013), the latter of which is more easily obtained and therefore used in this analysis.

4.1 Period estimation from instrumental response

Spectral ratios can be determined as the ratio of the spectral acceleration of the seismic response recorded at a given location in the structure, $SA_n(T)$, and the spectral acceleration recorded at a reference location, $SA_0(T)$ (often the ground floor or basement level):

$$SR_n(T) = \frac{SA_n(T)}{SA_0(T)} \tag{1}$$

By plotting the spectral ratios as a function of vibration period, T, the characteristic vibration periods of the structure can be ascertained as illustrated subsequently. Inspection of the spectral ratios for the instrumented structures (Figs. 1-5) reveals that the non-dimensional acceleration peaks occur at generally the same period for all spectral ratios generated for a given building (i.e. using different instruments for $SA_n(T)$). However, some of these peaks occur over a wide period range and therefore assigning a single fundamental period can be problematic. The technique used in these scenarios was to identify the period at which the majority of peaks from the various instruments and events coincided and then adopting this as the fundamental period estimate as reported in Table 3. The displacement time histories for each structure were compared for multiple instruments to ensure the building moved in phase indicating that the period determined was that of the first translational mode.

It can be seen that some of the spectral ratios exhibit additional peaks at approximately one third of the fundamental period, specifically in Figures 1b, 1d, 3a and 3c. This is indicative of the period of the second mode although clearly it is not as easy to resolve as the fundamental mode based on the data quality and quantity available. The general shape of the spectral ratios for different levels of the structure remains constant for the majority of structures.

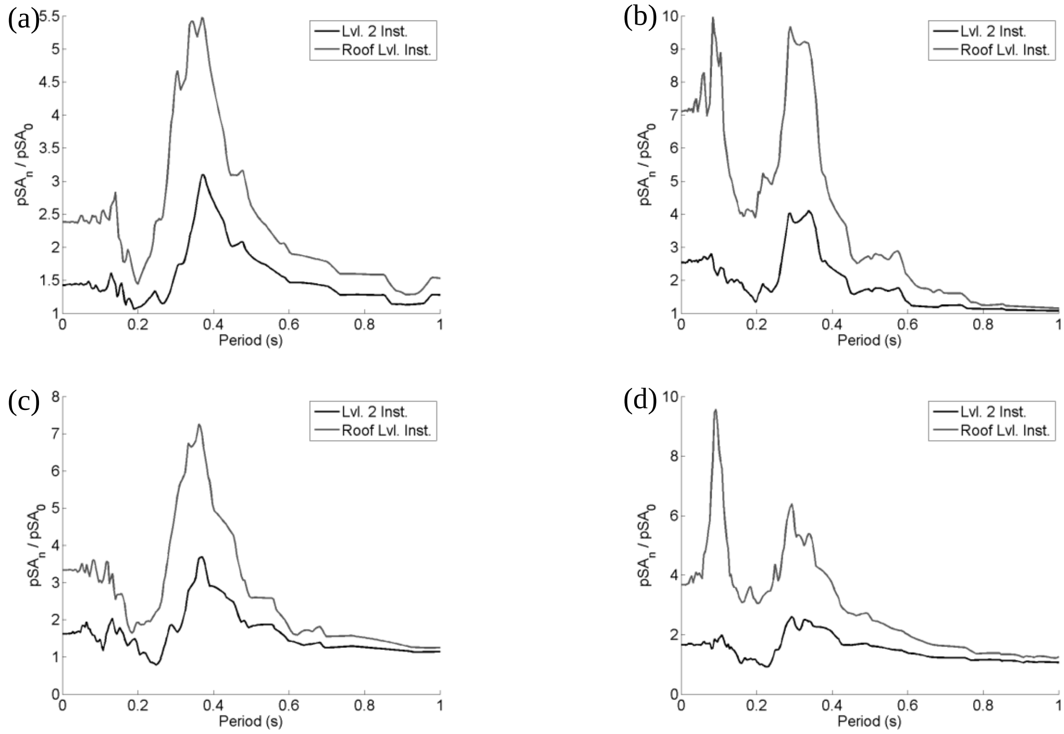

Figure 1. Period determination for Unit 2 of the GNS Science building using spectral ratios from multiple instruments: 16th August 2013 M_w 6.6 event in (a) transverse and (b) longitudinal direction, 21st July 2013 M_w 6.5 event in (c) transverse and (d) longitudinal direction.

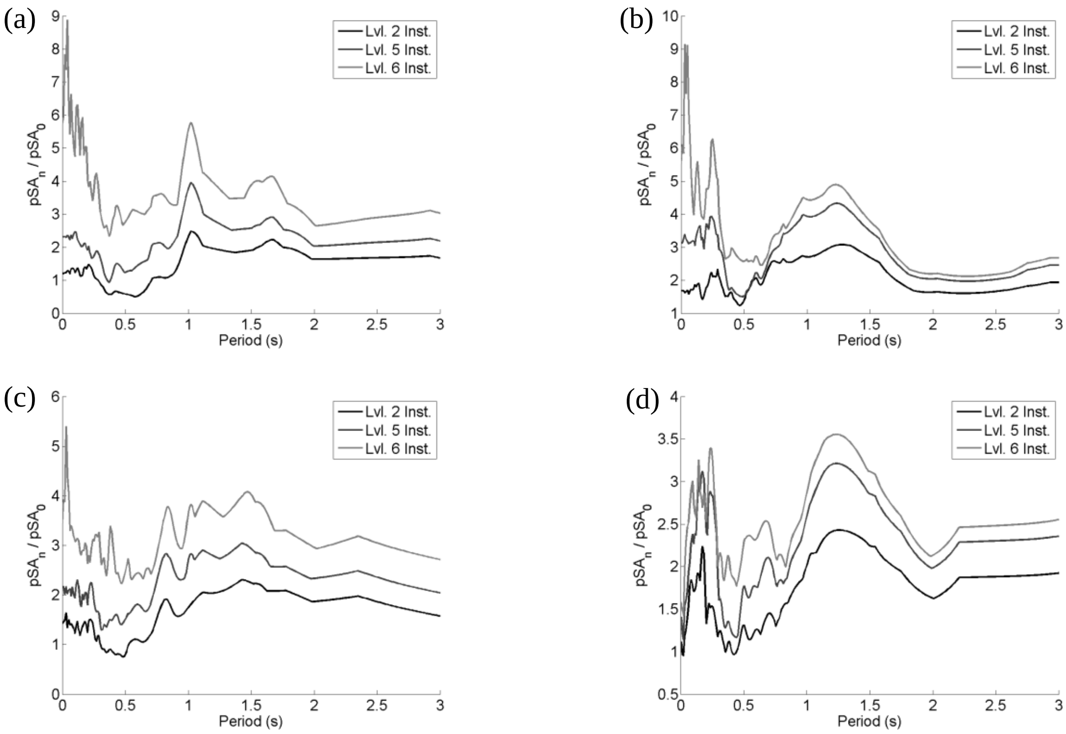

Figure 2. Period determination for the BNZ Centre Port building using spectral ratios from multiple instruments: 16th August 2013 M_w 6.6 event in (a) transverse and (b) longitudinal direction, 21st July 2013 M_w 6.5 event in (c) transverse and (d) longitudinal direction.

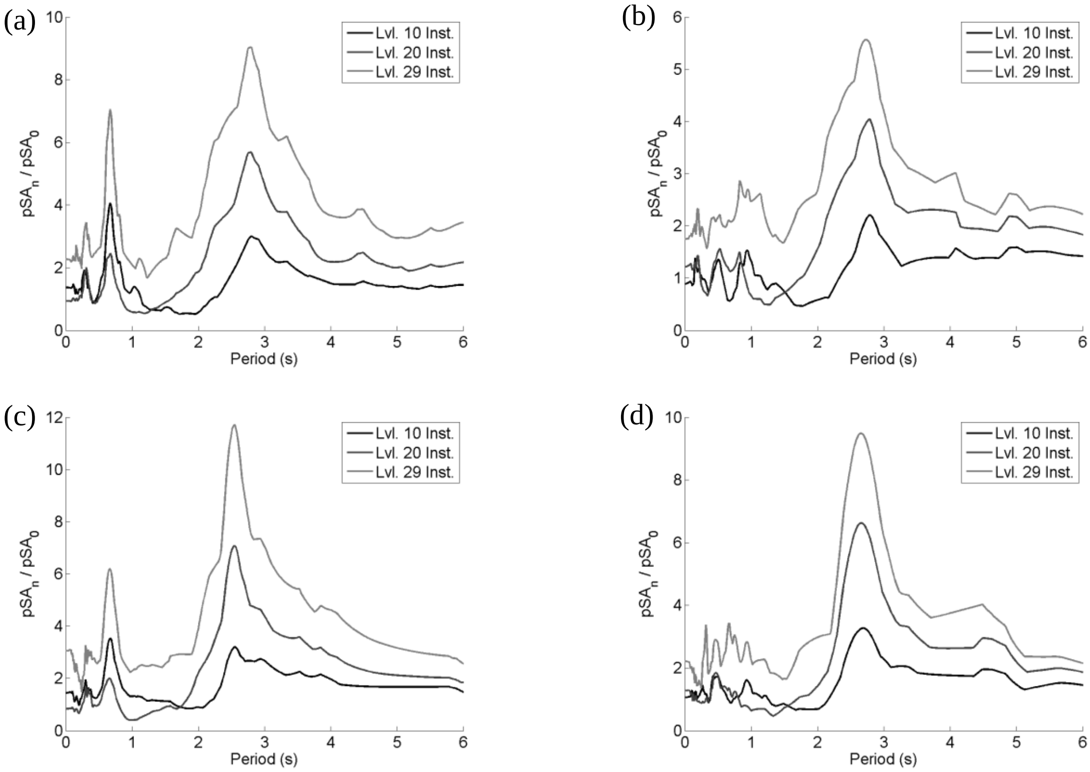

Figure 3. Period determination for Majestic Centre using spectral ratios from multiple instruments: 16th August 2013 M_w 6.6 event in (a) transverse and (b) longitudinal direction, 21st July 2013 M_w 6.5 event in (c) transverse and (d) longitudinal direction.

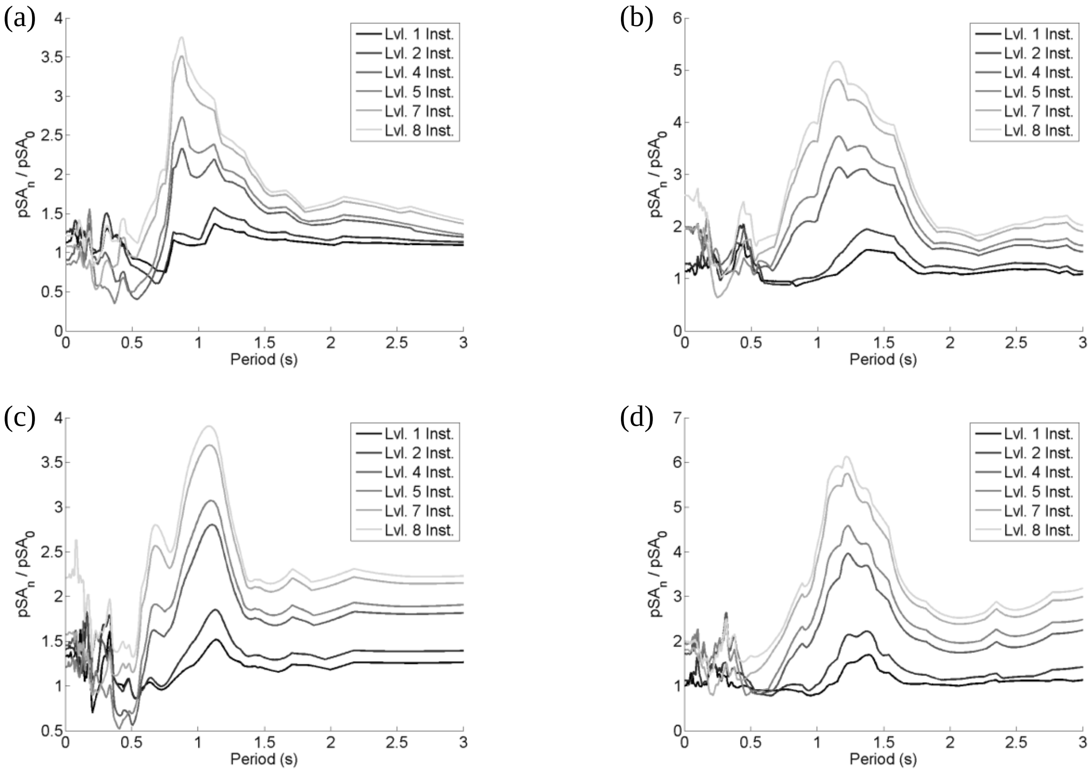

Figure 4. Period determination for the Victoria University Te Puni Village Tower using spectral ratios from multiple instruments: 16th August 2013 M_w 6.6 event in (a) transverse and (b) longitudinal direction, 21st July 2013 M_w 6.5 event in (c) transverse and (d) longitudinal direction.

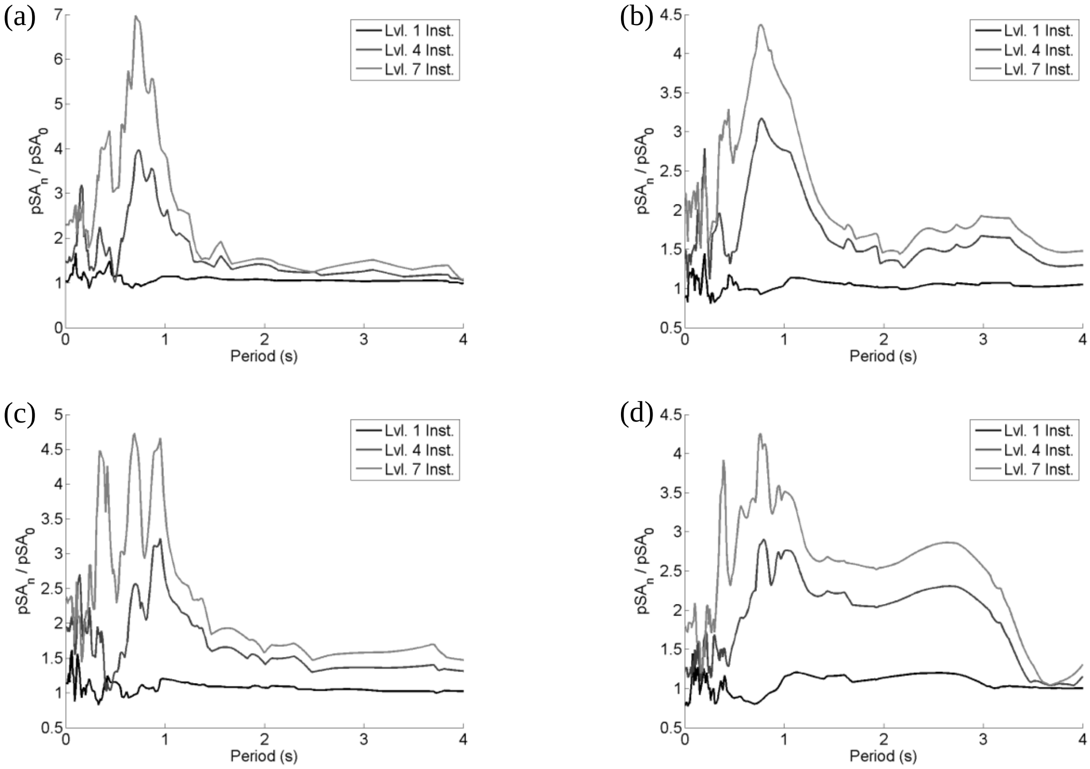

Figure 5. Period determination for Wellington Hospital using spectral ratios from multiple instruments: 16th August 2013 M_w 6.6 event in (a) transverse and (b) longitudinal direction, 21st July 2013 M_w 6.5 event in (c) transverse and (d) longitudinal direction.

4.2 Comparison of observed vibration periods with simplified predictions from NZS1170.5:2004

A simplified method for period estimation proposed in the New Zealand Loadings Standard (Structural Design Actions Part 5: Earthquake actions NZS1170.5, 2004) can also be used to estimate the fundamental period of a structure, which for the serviceability limit state (SLS) is given by:

$$T_1 = k_t h_n^{\ 0.75} \tag{2}$$

where k_t is a lateral load resisting factor, and h_n is the height in meters from base to uppermost seismic mass.

Equation 2 was used to compute the period for each of the analysed structures and is shown in Table 3. It is noted that the code equation is only a function of height and the lateral load resisting system, therefore the resulting estimation of the fundamental period applies in both lateral directions. Table 3 illustrates that the period predicted by Equation 2 differs from the instrumentally observed period by up to 65% in some cases. However for the Wellington hospital the code period approximation differs by less than 10% for both lateral directions.

It is important to note that Equation 2 is intended for use with force-based design, and as such should generally provide an under-estimation of the fundamental period to ensure that the spectral acceleration and subsequently the seismic design forces are conservatively estimated. Figure 6 illustrates how the estimated (NZS1170.5, 2004) and observed (spectral ratio) fundamental periods relate. It can be seen that for the Victoria University and GNS Science structures the equation provides an over-estimation of the period (producing an under-estimate of the design forces), while for the CPLB structure the period is under-estimated.

Table 3. Fundamental periods of analysed structures determined through spectral ratios and empirical code equation for estimating period.

Building name (code)	Transverse (s)	Longitudinal (s)	NZS1170.5 (2004) (s)
GNS Science Avalon (AVAB)	0.37	0.31	0.51
BNZ Centre Port (CPLB)	1.3	1.3	0.89
Majestic Centre (MJCB)	2.7	2.7	*
Victoria University Te Puni Village (VUWB)	1.0	1.3	1.6
Wellington Hospital (WHSB)	0.95	0.89	0.96

*Equation 2 is not applicable for this structure due to its height.

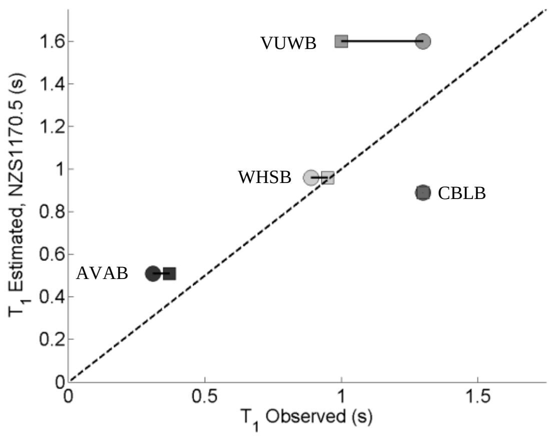

Figure 6. Comparison of estimated and observed fundamental periods in the longitudinal (square) and transverse (circle) directions.

5 PEAK FLOOR ACCELERATION DEMANDS

The majority of the direct monetary losses due to a large earthquake are associated with the damage of non-structural components and contents. One of the principal causes of damage to these components are large earthquake induced accelerations and therefore the accurate quantification of these demands can significantly reduce the economic impact of a ground motion event.

Current seismic provisions generally prescribe the acceleration demands as a function of normalized height and the PGA at the base of the structure. The code prescriptions in the American Society of Civil Engineers (Minimum Design Loads for Buildings and Other Structures ASCE/SEI7-10, 2010) estimate the accelerations as a linear distribution between one and three times the PGA from the base to the uppermost level of the structure, while the provisions in NZS1170.5 (2004) dictate a trapezoidal distribution over the lower 12m or 20% of the structure ranging from one to three times the PGA with a linear distribution of three times the PGA over the remaining height. It is critical to note that both of these prescriptions fail to account for the structure-specific fundamental period, damping ratio, and type of lateral load resisting system, all of which strongly affect the floor level acceleration demands (Miranda & Taghavi, 2005).

Figures 7-11 illustrate the observed maximum acceleration demands in both lateral directions for the five structures considered. It can be seen that for a given building and direction the profile of acceleration demands is similar for each event. As already noted, this is indicative of the fundamental properties of the structure (period, lateral load resisting system and damping) controlling the acceleration demands. Three of the five buildings (GNS Science, BNZ Centre Port and the Majestic Centre) exhibit acceleration demands in excess of the both code prescriptions, while the other two structures (Victoria University and Wellington Hospital) exceed the ASCE/SEI7-10 (2010) but not the

NZS1170.5 (2004) provisions.

The acceleration demands the Majestic Centre (Fig. 9) exhibit clear higher mode effects (reflected by large acceleration demands in the top floors). This is once again indicative of the fundamental structural properties controlling the acceleration demand profile and is a clear indication of the deficiencies of the current code prescriptions in estimating the acceleration demands.

A simplified method for estimating the acceleration demands has been proposed by Miranda and Taghavi (2005). This method utilizes the fundamental period, lateral load resisting system and damping ratio of the structure to estimate the acceleration demand profile, and has been recommended to replace current code prescriptions. The method has been used to accurately predict the acceleration demands in an instrumented building for multiple events (Thomson, 2013). Other methods for estimating the floor level accelerations have been proposed. However these have not yet been incorporated into current standards.

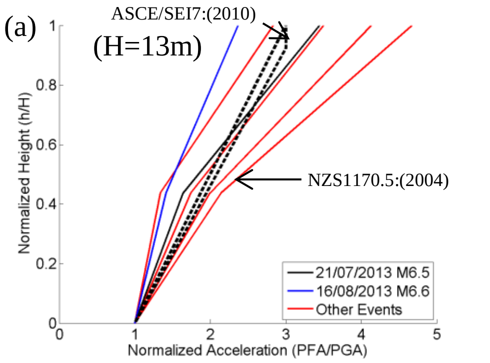

Figure 7. Measured acceleration demands compared with code prescriptions for Unit 2 of the GNS Science building in: (a) the transverse and (b) the longitudinal direction.

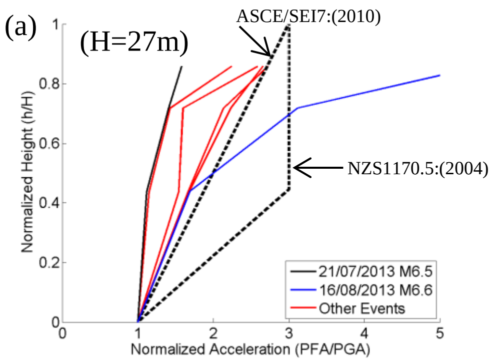

Figure 8. Measured acceleration demands compared with code prescriptions for the BNZ Centre Port Building in: (a) the transverse and (b) the longitudinal direction.

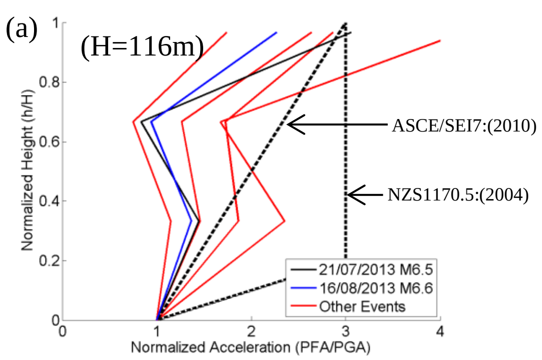

Figure 9. Measured acceleration demands compared with code prescriptions for the Majestic Centre in: (a) the transverse and (b) the longitudinal direction.

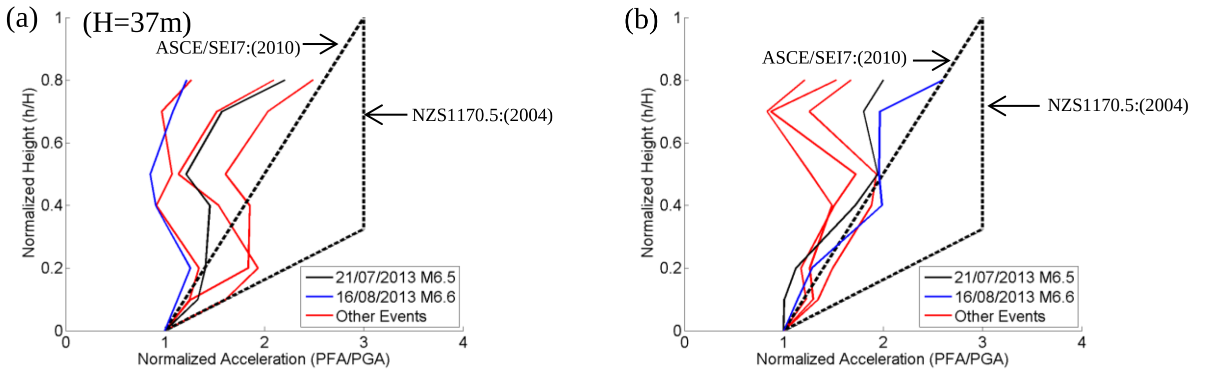

Figure 10. Measured acceleration demands compared with code prescriptions for the Victoria University Te Puni Village Tower in: (a) the transverse and (b) the longitudinal direction.

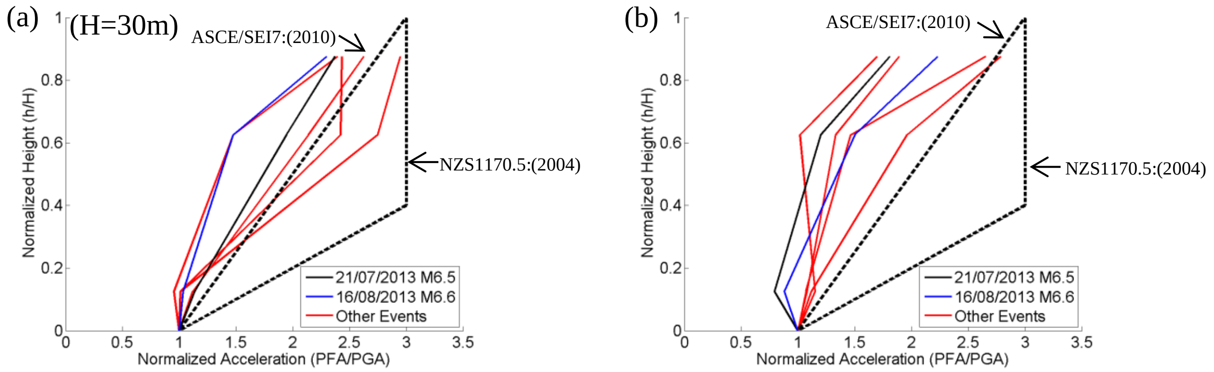

Figure 11. Measured acceleration demands compared with code prescriptions for Wellington Hospital in: (a) the transverse and (b) the longitudinal direction.

6 CONCLUSION

This paper has presented a preliminary analysis of the dynamic response of five instrumented buildings in the greater Wellington region for five ground motion events. For four applicable structures, the periods determined through spectral ratios were compared with those derived from an empirical equation prescribed in NZS1170.5 (2004). It was found that for two of the four of cases the code based equation is unconservative for use in forced-based design, while for one structure it was notably conservative. Additionally, the acceleration demand profiles were analysed and contrasted against two code prescriptions which were found to be inadequate for predicting the acceleration demands due to their failure in utilizing the fundamental structural properties.

ACKNOWLEDGEMENTS

The instrumental data utilized in this study were obtained as part of the GeoNet Building Instrumentation Programme. Assistance from Kevin Fenaughty and Caroline Little (GNS Science) in obtaining this data is greatly acknowledged. Financial support of this research from the University of Canterbury's Summer Scholarship Scheme and the New Zealand Earthquake Commission (EQC) are greatly appreciated.

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